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CYCLIC TENSILE TESTING OF A PILE IN GLACIAL TILL

R.P.L. McAnoy
A.C. Cashman
D. Purvis

Taylor Woodrow Research Laboratories,
345 Ruislip Road,
Southall,
Middlesex
England

RESUME

For offshore structures in deep water, piles are being designed to withstand cyclic tension, due to uplift and wave loading, throughout their design life. However there is a scarcity of data concerning the load levels which can be safely applied in this manner. This paper reports tensile tests on a heavily instrumented 10 metre long pile jacked into glacial till. Previous work had shown satisfactory pile behaviour under cyclic tensile loads peaking at up to 48% of the ultimate tensile capacity, and so this work was aimed at investigating pile response to more severe load levels, approaching failure. Cyclic tests were performed with varying peak loads up to 80% of the initial static capacity, and up to 13,500 cycles were applied depending on pile response. The pile sustained encouragingly high loads without serious deformation, but failure did occur during the most severe test, when the peak load was nominally 80% of the ultimate tensile capacity. Pile response analysis provided insight into compression pile design methods when applied to tension piles. Alpha and Lambda methods, but not the Beta method, estimated ultimate tensile capacity well, whilst stiffness was greater than implied by published T-Z curves.
1.0 INTRODUCTION

The oil and alternative energy industries, by their increasing interest in large scale floating structures, have stimulated research into the behaviour of conventional steel pipe piles as anchors.

A major development programme has recently been completed for one such floating system, required to provide a working platform in the 200-400 metre range of water depths (Smith and Taylor 1980). The structure comprises a buoyant column, tethered to a foundation anchorage by a series of tendons. Due to the positive buoyancy of the column, the fluctuating loads imposed on the foundation are always tensile.

The choice for the foundation design was between a piled or gravity base. Most of the foundation development programme was directed towards a piled base.

Available design guidance was considered inadequate for both the ultimate carrying capacity of piles in tension and pile response to working loads. Further, the effect on pile capacity of the vertical and lateral cyclic loads transmitted during severe storms could only be tentatively quantified. With this level of uncertainty the preliminary design method had to ensure that relatively low stress levels were applied to the piles during the 100 year storm. To check the adequacy of the design an experimental programme of laboratory and onshore large scale pile tests was undertaken. These tests substantiated the design adopted, examined the stability of a soil during representative storm loadings and determined the relative efficiency of jacking force to tensile capacity. In these tests the basic pile parameters and loading regimes were applicable to a jacked pile arrangement in a preliminary foundation design.
On completion of this section of the field test programme it was felt that it would be of considerable value to determine the behaviour of a single pile under more severe loadings. Thus a single, heavily instrumented, open ended 10.4m long steel pipe pile was tested under a series of high level tensile load packages. The results of these tests are presented in this paper. Further details of the laboratory and field test programme have been reported by Garas (1979) and Garas and McAnoy (1980).

2.0 DESCRIPTION OF FIELD TESTS

2.1 TEST SITE

Taking into account likely North Sea locations for a buoyant column structure, the most suitable ground condition for an onshore test site was considered to be a clayey glacial till. The UK Building Research Establishment (BRE) had already established a test site on the Holderness coast at Cowden, 23 km north east of Hull, Humberside, to investigate the properties of glacial till. An area of this site was made available for these pile tests together with site investigation data, including large scale plate tests and pressuremeter results.

The ground consists of an exposed thick deposit of uniform overconsolidated, sandy silty clay till, with interposed layers of sand and gravel. Boreholes and cone penetrometer results adjacent to the test position indicate zonal weathering of the till down to 4.5 metres and a dense grey black sand and gravel layer at 10.5 metres. Marsland and Powell (1980) gave a more detailed description of the till. A comparison of the residual strength of this Cowden till with various other clay types has been presented by Lupini et al (1981).

Table 1 shows typical soils data as assessed from information provided by the BRE, and from the above references. Figure 1 (a) shows a typical borehole log for the site, adjacent to the test position and 1 (b) typical cone penetrometer readings.
2.2 TEST PILE AND INSTRUMENTATION

The test pile was one of four steel pipe piles (193mm diameter, 9mm wall thickness by 10.4m long) which were each jacked 9.9m into the ground to form a square group at 580mm centres. They were installed open ended with a cutting shoe to reduce plugging. External ducts were welded to the piles to protect the instruments and cables. The location of the instruments on the most heavily instrumented pile, used for these single pile tests, is shown on Figure 2.

The axial load along the pile was monitored by $\frac{1}{4}$ bridge weldable electrical resistance strain gauges, positioned as pairs, diametrically opposite each other. The imposed head loads were monitored by a load cell built into the loading system. Pore water and total lateral soil pressure measurements at the pile/soil interface were taken by miniature electrical transducers mounted in the ducts, flush with the outer face.

Pile head vertical and horizontal displacements were measured relative to an independent reference beam by linear displacement transducers. Pore pressures in the surrounding soil at 2, 6 and 12 radii and 3.5m depth were measured by hydraulic piezometers open to the atmosphere.

All instruments were calibrated and checked in the laboratory, to ensure as far as possible that they would perform satisfactorily during testing. Whenever practical, the same calibration procedure was repeated on site before pile installation.

2.3 TEST PROGRAMME

The four piles were jacked into the ground over an eight day period in April 1980. A pile was pitched and the instrumentation checked during one day, and the pile installed the next. On average installation took 6 hours to reach the full embedded length of 9.9 metres.
Over a 41 day period from the end of jacking, the measured excess pore pressure at the pile-soil interface reduced to within 5% of the hydrostatic pressure. During this period the test facility was rearranged to accommodate the load actuating system, and a rigid steel pile cap was welded to the head of the piles, leaving a clear gap between the pile cap and the ground.

A series of cyclic load tests, summarised in Figure 3, were then carried out on the pile group. This test work, which is to be discussed by McAnoy et al. (1982), included a load package equivalent to the spectrum of a severe storm, with applied loads of up to 48% of the vertical or 6% of the horizontal estimated pull out capacity \( U_t \). The estimate of \( U_t \) was based upon the capacity of a single pile of identical dimensions installed and tested on the same site adjacent to the test position during the summer of 1979.

At the conclusion of the group tests it was decided that it would be of considerable value to determine individual pile performance under even more demanding loading conditions. The pile cap was therefore dismantled, and the loading system positioned over one of the four piles. A constant rate of extraction test to failure was carried out as soon as possible after the group tests to determine the ultimate capacity of this pile. The value obtained was then used as the initial capacity of the single pile prior to high level cyclic loading. Results from this initial extraction test and subsequent high level cyclic loading of this pile have been used for the analysis presented. A summary of the test programme considered is shown in Figure 3(b).

Each of the three cyclic tests shown was immediately followed by a constant rate of extraction test to determine the ultimate pullout capacity. A rate of displacement of 0.75 mm/min was used, as recommended by Whitaker (1963) for piles in cohesive soils. The tensile loads and numbers of cycles applied were nominally:

<table>
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<th>Mean</th>
<th>Cyclic Component</th>
<th>No. of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>40% U_A</td>
<td>± 20% U_A</td>
<td>11,775</td>
</tr>
<tr>
<td>40% U_B</td>
<td>± 40% U_B</td>
<td>564</td>
</tr>
<tr>
<td>30% U_C</td>
<td>± 30% U_C</td>
<td>13,597</td>
</tr>
</tbody>
</table>
for tests A, B and C respectively, where $U_A$ is the ultimate capacity determined immediately before test A, etc.

Sinusoidal loading was performed at two frequencies ($0.1H_z$ and $1.0H_z$) to maintain similitude of strain and drainage respectively with a prototype pile. Problems of similitude for scaled pile tests are discussed by Gallagher and St. John (1980). To ensure that viscous damping did not affect cyclic displacements when drainage was being modelled, testing was performed under displacement control for the higher frequency.

3.0 ANALYSIS OF FIELD TEST DATA

3.1 ULTIMATE TENSILE CAPACITY

3.1.1 Results

Figure 4 shows the six constant rate of extraction tests to failure performed on the pile. The first two tests E1, E2, were carried out before the pile had been subjected to high level cycling and were used to deduce the ultimate tensile capacity. The change in the capacity after applying the cyclic load was assessed by using the capacity recorded on extraction tests E3 to E6.

The maximum load from test E1 was 495kN, which was comparable to 481kN measured during test E2. The first test, E1, was carried out a month after the completion of the group tests, and the second test, E2, was performed a month later, one day before the start of the cyclic tests. The slopes of the load displacement curves are however considerably different. The first curve was non-linear over the majority of its length and peak capacity was reached at 5.2mm displacement. The second curve was linear up to over 80% of the ultimate capacity, which was reached at 3.2mm displacement.

Extraction test E3, performed immediately after cyclic test A, indicated a decrease in ultimate load capacity, to 457kN. Extraction test E4, after cyclic test B, indicated a further decrease to 400kN. The
last two extraction tests, E5 and E6, showed a slight increase of ultimate capacity. The shape of the load-displacement curves are linear over 80% of their range. The corresponding displacement required to mobilise these values were reasonably consistent at about 2.5mm.

3.1.2 Predictions of Ultimate Tensile Capacity

Three conventional methods of predicting ultimate pile capacity have been assessed; the Alpha, Beta, and Lambda methods. None of these methods distinguish between tensile and compressive loading.

In using them a number of assumptions have been made based upon site investigation data and earlier results of pile load distribution. Firstly it is assumed that the uppermost 1.0m of the ground contributed nothing to the side resistance, since strain gauge readings indicated that resistance was very low in this region. Secondly the Cu vs depth profile can be idealised to 170 kN/m² for depths 1.0m to 4.5m and 120 kN/m² from 4.5m to 10.0m depth, as suggested by pressuremeter test results (Table 1).

Alpha Method

API RP2A (1980) recommends that this total stress method is to be used for soils of medium to low plasticity and can be generally applied to overconsolidated clays. It is assumed that the ultimate skin friction on the pile wall is proportional to the undrained shear strength, according to the equation.

\[ \tau_s = \alpha C_u \]

where

- \( \tau_s \) = ultimate shear stress on pile wall
- \( \alpha \) = 0.5 for the values of \( C_u \) assumed above.

The alpha method predicted a static tensile capacity of 410 kN.
Beta Method

A number of methods have recently been developed using the effective stress parameters $c', \phi'$. These methods generally require an estimate of the radial stress on the pile shaft after set up. In heavily overconsolidated clays this is difficult to obtain.

The basic effective stress equation involving $\beta$ as suggested by Burland (1973) is;

$$\tau_s = \beta \sigma_{\text{vo}}'$$

where

$\beta = K_s \tan \phi'$

$\phi' = \text{softened drained angle of shear resistance}$

$K_s = \text{coefficient of lateral earth pressure}$

$\sigma_{\text{vo}}' = \text{insitu vertical effective stress}$

It is assumed throughout that $c' = 0$ due to disturbance of the soil during installation.

Meyerhof (1976) has analysed a large number of pile tests on bored and driven piles to derive $\beta$ values. He defined $\beta$ as above but from his work states that $K_s$ varies from roughly $K_o$ to 2 $K_o$ for driven piles, and suggests that on average $K_s$ may be taken as 1.5 $K_o$ in stiff clays. An empirical relationship between the overconsolidation ratio, $\phi'$, and $K_o$, is also suggested:

$$K_o = (1 - \text{Sin} \phi') \sqrt{\text{OCR}}$$

where

$K_o = \text{coefficient of earth pressure at rest}$

$\text{OCR} = \text{overconsolidation ratio}$

Hence

$$K_s = 1.5 (1 - \text{Sin} \phi') \sqrt{\text{OCR}}$$

Meyerhof's formula predicted an ultimate pile capacity of 270kN.

Lambda Method

The Lambda method (Vijayvergiya & Focht, 1972) has been developed from the same pile test data as the alpha method, but incorporates the
effect of penetration length on ultimate skin friction, which is calculated from the equation

\[ U_T = \lambda (\sigma'_v + 2 C_u) A_s \]

where
- \( \lambda \) = dimensionless constant, which is a function of pile penetration
- \( A_s \) = surface area of pile
- - = signifies average over pile length

The Lambda method predicted an ultimate pile capacity of 430 kN.

### 3.1.3 Discussion on Ultimate Capacity

A summary of measured and predicted ultimate capacities is given in Table 2. Two of the extraction tests have been used to highlight the measured capacities before and after failure under cyclic loading (Tests E1 & E4 respectively). It is possible that the 19% change in measured capacity during this series of tests was influenced by the formation of continuous slip surfaces, as 18mm of cumulative displacement was recorded. The reduction in capacity was close to that which could be expected if the effective angle of friction of the soil reduced to the residual.

The alpha and lambda methods provided reasonable estimates of the ultimate tensile capacity, and compared particularly well with the measured capacity after cyclic failure. In order to have predicted the measured capacity before cycling the alpha values used would have had to be approximately 0.6. Previous case studies indicate that alpha is often higher than the recommended value of 0.5. Further it is generally expected that jacked piles have a greater capacity than driven piles. The greater measured capacity may also reflect the influence of the installation of the adjacent three piles due to a change of insitu stresses acting on the pile shaft.

The beta method provided poor correlation between measured and predicted results. However, Meyerhof (1976) does show a wide scatter of \( \beta \) values from case records of piles in stiff clays. It has been
suggested that part of the reason for this lies in the values of the effective stresses assumed (Parry & Swain, 1977), as it is difficult to assess the correct values. This uncertainty is reflected in the poor correlation noted above. However, given a better determination of the effective stresses to be used and the manner in which they change due to installation and loading, the method could in principle allow for the effects of adjacent piles, method of installation and direction of loading. Better correlation has been achieved in soft clays, in which effective stress can be more accurately assessed.

Difficulties were experienced in assessing the undrained shear strength values to be used in the calculations as the various insitu and laboratory test methods indicated very different values. Most methods indicated that for 0-10 metre depth the top 4.5 metres of soil had a higher $Cu$ value than the soil below. However, the magnitude of the difference varied considerably.

3.2 RESPONSE DURING CYCLIC TESTS

Displacements during tests A, B and C are shown in Figures 5 and 6. Tests A and C produced surprisingly small permanent displacements - 0.12 and 0.14mm respectively - after over 11,000 cycles despite the high load levels applied. A linear relationship between mean displacement and log number of cycles is evident, with the gradient of the line being slightly greater for A than for C, reflecting the difference in peak loads. During the first 40 cycles or so, test B exhibited a similar linear relationship, although the gradient of the line was considerably greater than for the other tests. However, from 40 cycles onwards an entirely different response occurred. Mean displacements increased drastically, and continued to do so up to the point where the change of mean displacement per cycle was increasing with every cycle, indicating pile failure. Once this point had been clearly passed test B was terminated - only 564 cycles had been necessary.
No change in cyclic displacement occurred within any of the three tests, indicating that there was no degradation of cyclic soil modulus, contrary to the suggestion of Poulos (1980) that cyclic shear degradation was a function of stress level and number of cycles. It is particularly surprising that no degradation in cyclic stiffness occurred during test B, especially when the pile was failing as is clearly shown in Figure 6(a).

3.2.1 Axial Load Distribution

Figure 7 shows the axial load down the pile under cyclic and static loading at the beginning and end of tests A, B and C. There is no significant change with number of cycles, indicating that no load shedding occurred during cycling. Furthermore there is no change in the relative proportion of load taken by each element during these cyclic tests and the ultimate values given by extraction test E3. It is clear that the average shear stress, represented by the rate of decrease of axial load with depth, varies considerably from the 1-3m depth to the 3-8m depth, which is broadly consistent with the change in soil strength at about 4-5m below ground, as discussed in Section 3.1.2.

3.2.2 Pore Pressures

Readings of excess pore pressure at the pile/soil interface, taken at three depths as shown in Figure 2 during tests A, B and C showed that initial loading gave rise to a small excess pore pressure. A maximum increase of 15 kN/m$^2$ was measured at 3.15m level. For tests A and C, cyclic loading did not lead to any build-up in excess pore pressure and the initial pressure reduced. There was some evidence to suggest a slight increase in excess pore pressure during the first 40 cycles of test B to a maximum of 20 kN/m$^2$ at the 3.15m level. As pull-out increased this pressure dissipated, and at the end of all three tests excess pore pressures were continuing to decrease. None had quite returned to zero, but the maximum value at the end of cycling was down to approximately 5 kN/m$^2$. 

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3.2.3 Pile Head Stiffness and Apparent Soil Shear Modulus

Values of pile and soil stiffness deduced from the cyclic tests are shown in Figure 8. The initial static stiffness was reasonably consistent between test A, B and C, whilst the final stiffness was similar for tests A and C but dropped by 12% in B. The effective stiffness reduced as the pile gradually pulled-out, the drop being drastic during test B.

The cyclic stiffnesses remain constant throughout each test, but vary from one test to another. Figure 9 shows a plot of pile head stiffness versus load range (i.e. maximum minus minimum head load) for the three cyclic tests. It appears that load range and hence stress range, has been the dominant factor in terms of cyclic stiffness, reflecting the non-linearity of the soil stress-strain curve, particularly at high load levels. Mean load level and rate of test would also be expected to influence cyclic stiffness, but are not thought to be significant in this case. For example, the highest loading rate, which occurred during test B, would be expected to produce the highest cyclic stiffness, when in fact test B exhibits the most flexible response.

The apparent shear modulus of the soil at the pile/soil interface has been derived for an element of soil at 4.5m depth and exhibits the same trend as pile head stiffness (Figure 8). The apparent shear moduli derived, which range from 50-76 MN/m², are 3 to 7 times greater than values determined from plate and pressuremeter tests, Table 1. Similar behaviour was observed by Gallagher & St. John from pile tests on the same site. This is to be expected as the shear strain during the tests was considerably lower than those produced by most insitu testing.

3.3 NUMERICAL ANALYSIS OF LOAD-DISPLACEMENT BEHAVIOUR

One method of estimating the load-displacement behaviour of a pile is to use a discrete element technique. The soil is modelled as a series of non-linear ground springs, characterised by 'T-Z curves' relating pile displacement to mobilised shear stress. The pile is divided into elastic elements acted on by the relevant ground spring. A beam-column computer program can then be used to determine the pile head
load-displacement curve. Results from the extraction tests have been analysed to derive experimental T-Z curves. Results from extraction test E4, which produced the lowest ultimate capacity, were considered to be of greatest interest, and are presented here.

3.3.1 Derivation of T-Z Curves from Experimental Data

The pile was divided into seven elements as shown in Figure 10. Using the measured drop in axial load across a given element, the average shear stress on that element was computed. Displacement, z, of an element was found from the measured pile head displacement and the axial load down the pile;

\[ z = Z_t - \frac{x}{A} \int_0^x \frac{Q(x)}{AE} \, dx \]

where
- \( Q(x) \) = axial load in the pile at any depth x
- \( A \) = sectional area of pile
- \( E \) = elastic modulus of pile material
- \( Z_t \) = pile head displacement.

3.3.2 Results and Predictions

From extraction test E4, the development of shear stress on each element for four load levels (a) - (d) are shown on Figure 11. Load level (d) represents the approximate ultimate capacity. The corresponding displacements of each element are also shown. Using information of this type the T-Z curves for elements 3-6 have been plotted on Figure 12. Also shown are 'predicted' curves, which have been produced from the work by Coyle & Reese (1966), who suggested a series of T-Z curves for piles installed in cohesive soils. Three standard curves were proposed for varying soil depths, each being presented in terms of undrained shear strength, after a correction to take into account the effect of pile installation. The predicted curves of Figure 12 have been produced using the in-situ undrained shear strength profile assumed in Section 3.1.2.
Vijayvergiya (1977) developed a different approach from results of load tests on compression piles. It was suggested that the equation:

\[ T = T_{\text{max}} \left( 2 \sqrt{Z/Zc} - Z/Zc \right) \]

where

- \( T \) = average shear stress on an element
- \( T_{\text{max}} \) = ultimate average shear stress on an element
- \( Z \) = vertical displacement of an element
- \( Zc \) = critical displacement of an element required to mobilise \( T_{\text{max}} \).

could be used to define T-Z curves for cohesive soils up to failure. Furthermore Vijayvergiya suggested that \( T_{\text{max}} \) should be derived using conventional methods for axial capacity, as discussed in Section 3.1.3, and that 5-8mm is a typical range of \( Zc \) for piles of 300mm diameter and greater. Figure 13, produced from Figure 12 shows that measured values of \( Zc \) decrease with depth from a maximum of approximately 1.5mm (0.8% of pile diameter) to 0.4mm. Figure 14 shows the difference in shape between normalised plots of Vijayvergiya's equation, measured values, and Coyle and Reese's curve for depths greater than 6 metres.

3.3.4 Discussion

It is clear from Figures 12 and 14 that the T-Z curves exhibit a bilinear behaviour, and that the curve suggested by Vijayvergiya is inappropriate in this case. The curve for element 3 indicates a slightly softer response than the remainder, but this is considered to be due to the existence of a low radial stress over the upper part of this element. Each curve can be described by two parameters, \( T_{\text{max}} \) and \( Zc \).

The values of \( T_{\text{max}} \) predicted by Coyle & Reese for element 6 agrees with measurements, but curves for elements 3, 4 and 5 yield gross underestimates. It would appear that a method of predicting \( T_{\text{max}} \) from cone penetrometer results is desirable, since some correlation can be observed between the shape of the ultimate shear stress distribution of Figure 11(a), and the sleeve friction measurements of Figure 1(b).
With respect to critical displacement, the recommendations of Coyle and Reese, and Vijayvergiya, again significantly overestimate measured values. Neither of these methods incorporate the effect of diameter on $Z_c$, although the measured results appear to conform to the suggestion of 1% pile diameter as an upper value for $Z_c$ (Whitaker and Cooke, 1976) for the piles tested. The reduction of $Z_c$ with depth which was observed is implied by the curves of Coyle and Reese, and is possibly due to an increase in confining pressure with depth.

In any assessment of existing methods of analysis, it is important to note the conditions for which the methods were designed. The curves of Coyle and Reese and Vijayvergiya were suggested for piles under sustained compression, and may be affected by direction and rate of loading. On first impressions, therefore, it might seem inappropriate to compare the curves with results from a constant rate of extraction test, which might be expected to produce a stiffer response, particularly at high load levels. However, to put this into perspective the cumulative displacements recorded after the completion of cycling in Test A have been plotted on Figure 12. The pile was still under the mean load and this displacement is considered as an upper bound to the displacements which would have been recorded during sustained static loading over the same period of 6½ hours. The point still indicates that the element response is stiffer than that predicted by Coyle and Reese. However at higher load levels displacements due to creep are more significant and thus may approach or even exceed predicted displacements.

4. CONCLUSIONS

Effect of Cycling on Ultimate Tensile Capacity

1. The ultimate tensile capacity ($U_t$) of the pile reduced by 19% during the test programme. This reduction was close to that which could be expected if the effective angle of friction of the soil reduced from the remoulded to the residual value.
2. The lowest $U_t$ value occurred immediately after the test B, when the pile failed under cyclic loading, and the reduction was probably due to a combination of cyclic loading and cumulative displacements.

Accuracy of Compression Pile Methods for Determining $U_t$

3. Good correlation between measured and predicted $U_t$ values was achieved using the alpha and lambda methods, which erred by 10-20% on the conservative side before cycling, and were within 7% of the measured value after cyclic failure.

4. The beta method provided a gross underestimate before and after cycling.

Response During Cycling

5. The pile sustained encouragingly high levels of cyclic loading, peaking at up to 60% $U_t$, with permanent displacements of only 0.14mm after 11,000 cycles.

6. At a higher load level, peaking at nominally 80% $U_t$, a dramatic change in behaviour occurred after a small number of cycles, and failure followed.

7. The cyclic stiffness of the pile did not vary with number of cycles in any of the tests, even during failure.

8. No significant build-up of pore pressure due to cycling was observed.

9. No load shedding down the pile occurred during cycling, even at failure.
T-Z Curves

10. The relationship between mobilised shear stress \( T \) and displacement \( z \) on a pile element was linear for \( Z < Z_c \), the 'critical' displacement. For \( Z > Z_c \), \( T \) remained approximately constant.

11. Ultimate shear stress, \( T_{\text{max}} \), predicted by the T-Z method of Coyle and Reese underestimated measured values in the upper levels of the pile, apart from the top metre, which took no significant load.

12. The distribution of \( T_{\text{max}} \) with depth showed some correlation with core penetrometer sleeve friction results.

13. The critical displacement reduced with depth, possibly due to an increase in confining pressure.

14. Critical displacements were considerably less than the values suggested by Coyle and Reese. This may be partly due to the test pile being of a smaller diameter than those from which the design curves were derived.

Future Work

This investigation was limited to the tensile behaviour of jacked piles in one soil type. Considerably more research is necessary before efficient design methods can be relied on for dynamically loaded tension piles. This research should concentrate on the load transfer mechanism, creep and sensitivity of soils to dynamic loads.

Taylor Woodrow and the U.K. Building Research Establishment have proposed an extensive development programme to measure the performance of different types of piles under representative tensile axial and lateral loading conditions and to assess appropriate design methods. This will incorporate and extend both organisations' present work in this area. (Negotiations with potential sponsors for this project are well advanced, although further sponsorship is still being sought).
Acknowledgements
The Authors would like to thank the Directors of Taylor Woodrow for their permission to publish these results, the U.K. Building Research Establishment for the use of their test site and site investigation results, and all Research Laboratory personnel involved in the project.

LIST OF REFERENCES


McAnoy, Cashman and Williams. To be Published


Table 1. Typical Soil Properties 0-10.0m Depths

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
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* Calculated from the equation \( C_u = (P_L - P_{HO})/6.18 \), where 
  \( P_L \) = limiting pressure & \( P_{HO} \) = original horizontal stress in the ground.

⁺ Average G for 1% strain

⁰ Secant modulus over range \( P_{vo} \) to \( P_{vo} + \frac{1}{3}(q_u - P_{vo}) \), where 
  \( q_u \) = ultimate base pressure & \( P_{vo} \) = original vertical stress in the ground.
Table 2. Summary of Ultimate Tensile Capacity

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Measured Value (kN)</th>
<th>Predicted Values (kN) and Predicted/Measured Values (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Alpha</td>
</tr>
<tr>
<td>E1</td>
<td>495</td>
<td>410</td>
</tr>
<tr>
<td>E4</td>
<td>400</td>
<td>410</td>
</tr>
</tbody>
</table>
FIGURE 1 (a) Borehole Log (b) Cone Penetrometer Log.
FIGURE 2 Pile Instrumentation
FIGURE 3. Load History of the Piles (a) Previous Loading During Group Test (b) Single Pile Test.
FIGURE 4. Constant Rate of Extraction Tests To Failure, E1–E6
FIGURE 5. Pile Head Displacements During Cyclic Tests A, B & C.
**FIGURE 6** Pile Head Displacements

(a) Cyclic Amplitude

(b) Mean
FIGURE 7 Axial Load Distributions During Testing.
<table>
<thead>
<tr>
<th>Description of Modulus</th>
<th>Test A</th>
<th>Test B</th>
<th>Test C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Static Loading</td>
<td>K</td>
<td>G</td>
<td>K</td>
</tr>
<tr>
<td></td>
<td>262 (252)</td>
<td>76</td>
<td>262 (252)</td>
</tr>
<tr>
<td>Cyclic</td>
<td>255 (215)</td>
<td>226 (198)</td>
<td>242 (233)</td>
</tr>
<tr>
<td>Effective, 10(^{th}) Cycle</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100(^{th}) Cycle</td>
<td>249</td>
<td>211</td>
<td>233</td>
</tr>
<tr>
<td>250(^{th}) Cycle</td>
<td>231</td>
<td>198</td>
<td>228</td>
</tr>
<tr>
<td>500(^{th}) Cycle</td>
<td>219</td>
<td>139</td>
<td>217</td>
</tr>
<tr>
<td>1,000(^{th}) Cycle</td>
<td>220</td>
<td>50</td>
<td>196</td>
</tr>
<tr>
<td>10,000(^{th}) Cycle</td>
<td>217</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Final Static Unloading</td>
<td>255</td>
<td>70</td>
<td>228</td>
</tr>
</tbody>
</table>

K - Pile Head Stiffness (kN/mm)
(K) - Stiffness, Derived From CRE Tests Over Same Load Range
G - Apparent Soil Shear Modulus at Mid-Depth of Pile (MN/m\(^2\))

FIGURE 8 Variations in Pile Head Stiffness and Apparent Soil Shear Modulus
FIGURE 9 Variation of Cyclic Stiffness With Load Range.
<table>
<thead>
<tr>
<th>Element No.</th>
<th>Depth to Centre (m)</th>
<th>Tensile Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.25</td>
<td>$T_1 = 0$</td>
</tr>
<tr>
<td>2</td>
<td>+0.5</td>
<td>$T_2$</td>
</tr>
<tr>
<td>3</td>
<td>+1.5</td>
<td>$T_3$</td>
</tr>
<tr>
<td>4</td>
<td>+2.5</td>
<td>$T_4$</td>
</tr>
<tr>
<td>5</td>
<td>+4.5</td>
<td>$T_5$</td>
</tr>
<tr>
<td>6</td>
<td>+7.0</td>
<td>$T_6$</td>
</tr>
<tr>
<td>7</td>
<td>+8.95</td>
<td>$T_7$</td>
</tr>
</tbody>
</table>

**FIGURE 10** (a) Division of Pile into Elements  
(b) Representation of T-Z Curves
FIGURE 11 (a) Distribution of Average Shear Stress & (b) Corresponding Elemental Displacements at Various Stages During Extraction E4
FIGURE 12 Comparison Between Measured and Coyle & Reese T-Z Curves.
Critical Vertical Displacement, $Z_c$, (mm)

FIGURE 13 Critical Vertical Displacement Versus Depth
FIGURE 14 Normalised T-Z Curves